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RELIABILITY-BASED METHOD FOR ASSESSING LIQUEFACTION POTENTIAL OF SOILS

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Abstract. *This paper explores probabilistic method for assessing the liquefaction potential of sandy soils. The current simplified methods for assessing soil liquefaction potential use a deterministic safety factor in order to determine whether liquefaction will occur or not. However, these methods are unable to determine the liquefaction probability related to a safety factor. A solution to this problem can be found by reliability analysis. This paper presents a reliability analysis method based on the popular certain liquefaction analysis method. The proposed probabilistic method is formulated based on the results of reliability analyses of 190 field records and observations of soil performance against liquefaction. The results of the present study show that confidence coefficient greater and smaller than 1 doesn't mean safety and/ or liquefaction in cadence for liquefaction, and for assuring liquefaction probability, reliability based method analysis should be used. This reliability method uses the empirical acceleration attenuation law in the Chalos area to derive the probability density distribution function and the statistics for the earthquake-induced cyclic shear stress ratio (CSR). The CSR and CRR statistics are used in continuity with the first order and second moment method [8] to calculate the relation between the liquefaction probability, the safety factor and the reliability index. Based on the proposed method, the liquefaction probability related to a safety factor can be easily calculated. The influence of some of the soil parameters on the liquefaction probability can be quantitatively evaluated.*

1 INTRODUCTION

Because of abundant earthquakes which occur in the world every year, numerous problems and damages are created. These damages are divided into two classes of structural and geotechnical damages. Structural damages (problems) are generally resulted from lack of attention and not observing technical principles during construction.

The other class is also resulted from due to geotechnical factors and influencing the building by local soil conditions and interaction between them witch liquefaction incidence, earth sliding, decrease in soil loading, subsidence and/or affecting by earthquake characteristics can be placed in this group. In liquefaction incidence, due to water pressure enhance, an opening is created and as a result of decrease in sandy soils resistance, it is saturated and soil takes a liquefied form which this phenomenon is as significant subsidence, eruption of mud and water and water and water leakage through earth (land) surface pores. [1]

It is very difficult to obtain high-quality intact samples and it costs high, today simple tests (experiments) which are performed in the place, are used which some tests including Standard Penetration Test (SPT), cone penetration Test (BPT), Intact penetration Test (CPT) and shear wave rate (speed) Test (Vs) are used. [2, 3]

In most projects, for obtaining liquefaction potential, Standard Penetration Test (SPT) obtained by seed and Idriss, is used. and cone penetration Test is less used, BPT test is used in Sandy and rubble stone soil.[4] In SPT method, limit conditions are usually (generally) used which separate liquefaction district from non- liquefaction district. These limit conditions (This limit condition) is generally obtained empirically and based on observations in the site which during these limit conditions, many of uncertainties are entered the calculation. While most potential assessment (evaluation) methods are based on crucial analysis which are not able to communicate exactly with liquefaction probability, due to the problems and high costs to prepare the intact and high- quality samples and also the presence of simple methods based on in-site (in- place) tests such as standard penetration test (SPT), geotechnical In most projects, for obtaining liquefaction potential, Standard Penetration Test (SPT) obtained by seed and Idriss, is used. and cone penetration Test is less used, BPT test is used in Sandy and rubble stone soil.[4] In SPT method, limit conditions are usually (generally) used which separate liquefaction district from non- liquefaction district. These limit conditions (This limit condition) is generally obtained empirically and based on observations in the site which during these limit conditions, many of uncertainties are entered the calculation. While most potential assessment (evaluation) methods are based on crucial analysis which are not able to communicate exactly with liquefaction probability, due to the problems and high costs to prepare the intact and high- quality samples and also the presence of simple methods based on in-site (in- place) tests such as standard penetration test (SPT), geotechnical engineers prefer generally these tests for liquefaction potential assessment (evaluation). One of the methods for liquefaction potential assessment (evaluation) is based on SPT (standard penetration test) which was developed by seed and Idriss. This empirical method for limit conditions extension enters crucially many of uncertainties into the calculation. In addition, most evaluation (assessment) methods are based on crucial analysis and don’t consider soil resistance changes and the loads resulted from earthquake and are not able to communicate exactly with liquefaction incidence probability [5].

2 RELIABILITY MODEL FOR SOIL LIQUEFACTION

Advanced first-order second-moment (AFOSM) techniques are used to calculate the reliability index in this study. Specifically, the Hasofer-Lind reliability index is computed as follows [6]:

$$\beta = \min_{x \varepsilon F} \sqrt{(X - m)^T C^{-1} (X - m)} \tag{1}$$

Where X = vector of random variables in the limit state function is given by G(X) = 0; m = vector of mean values; and C = covariance matrix. The minimization in (1) is performed over the failure domain F related to the region G(X) < 0. A number of numerical techniques have been used to solve this minimization problem. The ellipsoid method [7, 8] is used here to perform the minimization and determine the reliability index β [7]. Among the practical advantages of this method are: (1) the solution can be obtained by working in original, rather than in changed or reduced random variable space; (2) it is not necessary to provide or calculate partial derivatives of G(X); and (3) associate and non-normal variables are handled easily through transformations [6].

For a reliability analysis of the liquefaction potential, the limit state may be written as G(X) = CRR/CSR - 1 = 0, where CSR is the cyclic stress ratio that shows the loading burden by an earthquake, and CRR is the cyclic resistance ratio that represents the liquefaction resistance of soil. The period of CRR was first described by Robertson and endorsed by the National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils [9]. In the present study, CSR is calculated using Seed and Idriss [1, 4] formulation:

$$CSR = \left(\frac{\tau_{av}}{\sigma_v^{'}} \right) = 0.65 \left(\frac{a_{\max}}{g} \right) \left(\frac{\sigma_v}{\sigma_v^{'}} \right) \left(\frac{r_d}{MSF} \right) \tag{2}$$

Where $a_{\max}$ = peak horizontal ground acceleration generated by the earthquake; g = acceleration of gravity; σ_v = total vertical overburden stress; $\sigma_v^{'}$ = effective vertical overburden stress; r_d = stress reduction coefficient; and FMS = magnitude scaling factor.

The term r_d gives a relative repair for the flexibility of the soil profile. Seed and Idriss [10] provide a chart showing the mean and the range of r_d values against depth. Liao and Whitman [11] proposed an excellent access to the Seed and Idriss mean r_d values. The latter method, which was adopted during the NCEER workshop [9], is used in the present study. Note that the certainty with which CSR can be calculated decreases with depth when using mean r_d values in the calculations [9]. Since almost all field liquefaction data available are at shallow depths, where the uncertainty is smaller, r_d is treated as a nonrandom variable, although it is a function of depth.

The term FMS is used to correct the calculated CSR for earthquakes with magnitudes smaller or larger than 7.5. This variable is a function of earthquake magnitude (M). This term is required since Seed and Idriss' [10] simplified method was originally developed for an earth-

quake magnitude of 7.5. In this study, Idriss' new formula $\left(F_{MS} = 10^{2.24} / M^{2.56}\right)$ mention with Youd and Idriss [9] is accepted. With this formula, (2) can be rewritten as follows:

$$CSR = \left(0.65/10^{2.24}\right)\left(a_{\max}/g\right)\left(\sigma_v/\sigma_v^{'}\right)\left(r_d\right)\left(M^{2.56}\right) \quad (3)$$

The variables $a_{\max}$, M, σ_v and $\sigma_v^{'}$ in (3) are the amounts as random variables in the present study. These random variables are all assumed to follow normal distribution. In this study, the reported values of these variables are taken as the means. The coefficients of variation (COV) of the variables $a_{\max}$, M, σ_v, and $\sigma_v^{'}$ are calculated to be 0.15, 0.05, 0.10, and 0.15, respectively. The assessment of COV levels for $a_{\max}$ and M should not be confused with a general liquefaction risk analysis, where the seismic loads are also considered as uncertain, in which case the COV of $a_{\max}$ could easily reach 0.50 or higher [8] due to uncertain attenuation. The COV for σ_v is estimated based on the fact that the unit weight of soils normally falls in the 15–21 kN/m3 range, and thus the COV may be estimated as 0.10, considering the range as the average +/- 2 standard deviations. The COV for $\sigma_v^{'}$ is considered to be somewhat greater than that for σ_v because of the uncertainty in the ground-water table.

A COV of 0.15 for $\sigma_v^{'}$ is considered suitable. As with any geotechnical projects, the responsibility of assessing input uncertainties is on the engineer, and if the understand uncertainties in terms of COVs differ significantly from the foregoing COV values, some regulation to the solution received in the probabilistic analysis is warranted. The CRR may be calculated based on SPT-based methods [12] or the CPT-based methods.

The COV for CRR model uncertainty is assumed to be 0.10, which is estimated based on the standard error of the CRR prediction as deviated from those calculated by Robertson and Wide's method [13]. The communication among the input variables also needs to be considered in the reliability analysis. There is strong communication between σ_v and $\sigma_v^{'}$ between $a_{\max}$ and M, while the communication for all other pairs of variables is weak. The communication coefficient is calculated by using standard statistical methods [14]. Based on the available data, the communication coefficient between σ_v and $\sigma_v^{'}$ is determined to be about 0.95, and a communication coefficient between $a_{\max}$ and M is determined to be about 0.90. These values are used in this paper, while all other variables are assumed to be independent.

The database, including120 liquefied cases and 70nonliquefied cases, is taken from the literature [15]. For each case, the tabulated data include depth of the water table, depth of the liquefaction observation, and the four input variables—$a_{\max}$, M, σ_v and $\sigma_v^{'}$. Here, the reported values of the four input variables are taken as the average values, the afore-mentioned COV values are used, and all random variables are assumed to be normally distributed. The reliability index β is calculated with the following equation, derived from (1) and based on the foregoing communication assumption:

$$\beta^2 = \min_{G(X)=0}\left\{\begin{array}{l}\left[\frac{(x_1-m_1)^2}{\sigma_1^2}\right]+\left[\frac{(x_2-m_2)^2}{\sigma_2^2}\right]+\left[\frac{(x_3-m_3)^2}{\sigma_3^2}-2\frac{(x_3-m_3)(x_4-m_4)\rho_{34}}{\sigma_3\sigma_4}+\frac{(x_4-m_4)^2}{\sigma_4^2}\right]\\ \cdot\left(\frac{1}{1-\rho_{34}^2}\right)+\left[\frac{(x_5-m_5)^2}{\sigma_5^2}\right]+\left[\frac{(x_6-m_6)^2}{\sigma_6^2}-2\frac{(x_6-m_6)(x_7-m_7)\rho_{67}}{\sigma_6\sigma_7}+\frac{(x_7-m_7)^2}{\sigma_7^2}\right]\\ \cdot\left(\frac{1}{1-\rho_{67}^2}\right)\end{array}\right\} \quad (4)$$

Where x_1 = random variable $(N_1)_{60cs}$; x_2 = random variable σ_v ; x_3 = random variable σ_v' ; x_4 = random variable $a_{\max}$; x_5 = random variable M_w ; m_i = mean value of random variable x_i (i = 1, 5); σ_i = standard deviation of random variable x_i ; and ρ_{23}, and ρ_{45} = correlation coefficients between x_2 and x_3, and x_4, x_5. The reliability index β determined from (4) is subjected to the limitation of G(X) = 0. The calculated β values are grouped according to whether or not liquefaction actually occurred at the site. For a case in which a reliability index β has been calculated, the probability that liquefaction will occur would be:

$$P_1 = 1-\phi(\beta) \quad (5)$$

Figure 1 shows the communication between reliability index and probability of liquefaction as for equation 5.

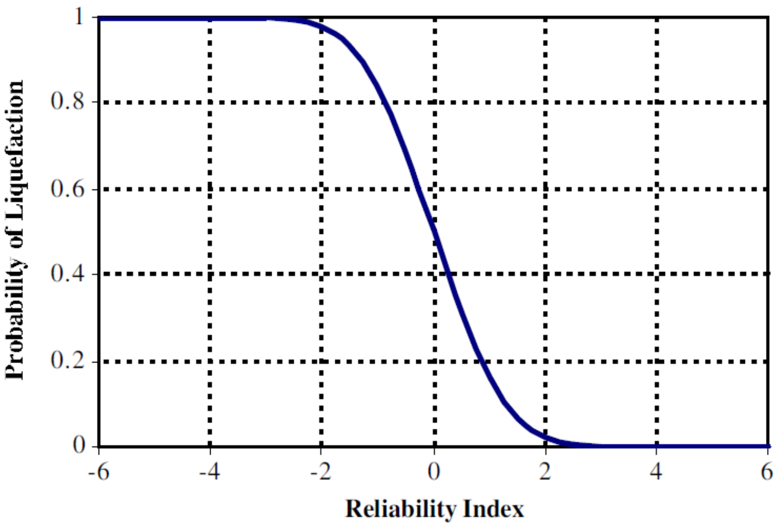


Figure 1: Communication between reliability index and probability of liquefaction [15]

3 CASE STUDY

In the present study, 190 case records from Chalos city have been analyzed. For each case, the CRR and CSR have been calculated, along with the reliability index and probability of liquefaction. Table 1 show a summary of this reliability analysis for all cases at the depths where soil performance against liquefaction was reported. For each of these cases, the CSR, CRR, and the probability of liquefaction (PL) have been calculated continuously at all depths so that a profile of PL could be drawn.

No.	Depth (m)	Soil type	P_L (%)	β	F.S	CSR	CRR	a_{max}	M_W	$(N_1)_{60CS}$	σ'_v T/m^2	σ_v T/m^2
1	2.5	SP-SM	81	-0.88	0.71	0.2	0.14	0.36	6.3	13	4.5	6
2	3	SP	87	-1.14	0.85	0.27	0.23	0.36	6.3	21	2.99	5.49
3	3.5	SP	54	-0.1	0.87	0.2	0.18	0.36	6.3	17	5.04	7.04
4	4	SP	82	-0.9	0.72	0.18	0.13	0.36	6.3	12	6.35	7.8
5	4.5	SM	77	-0.75	0.77	0.17	0.13	0.36	6.3	12	6.59	7.82
6	5	SP	97	-1.9	0.45	0.19	0.09	0.36	6.3	7	6	8
7	5.5	SP	27	0.62	1.08	0.19	0.21	0.36	6.3	19	7.5	10
8	6	SP-SM	99	-2.53	0.31	0.24	0.07	0.36	6.3	5	4.35	7.05
9	6.5	SM	26	0.65	1.09	0.2	0.21	0.36	6.3	20	5.59	11.82
10	7	SM	76	-0.7	0.78	0.2	0.15	0.36	6.3	14	9.5	13
11	8	SP	96	-1.8	0.43	0.22	0.1	0.36	6.3	8	9	14
12	10	SP-SM	32	0.5	1	0.22	0.21	0.36	6.3	20	13	19
13	11	SP-SM	94	-1.59	0.52	0.21	0.11	0.36	6.3	9	14	22
14	11.5	SP-SM	19	0.86	1.12	0.21	0.24	0.36	6.3	22	14	21.9
15	12	SP	56	-0.15	0.82	0.23	0.19	0.36	6.3	18	13	23.4
16	16	SP-SM	77	-0.73	0.74	0.19	0.14	0.36	6.3	13	19	31.9
17	16.5	SP-SM	38	0.3	1.03	0.15	0.16	0.36	6.3	15	22.7	32
18	17	SM	16	1	1.17	0.19	0.22	0.36	6.3	21	18.7	34.19
19	18	SP-SM	12	1.18	0.3	0.19	0.25	0.36	6.3	22	19	36
20	19	SM	50	0	0.85	0.2	0.36	0.36	6.3	16	15.1	33.6

Table 1: The geotechnical parameters of soils in the Chalos area

Figure2 shows a sample output of the PL profile, along with the CSR and CRR profiles and the input SPT profiles. The drawing of the CSR and CRR profiles, such as those shown in figure 3(b), are quite useful, as they show which layers are likely to liquefy. However, this assessment of the liquefaction potential is essentially deterministic. Because of the uncertainties involved in the calculation of CSR and CRR, such a deterministic approach is not always appropriate. The drawing of the PL profile, as shown in figure2 (d), offers an alternative on which engineering decisions may be based.

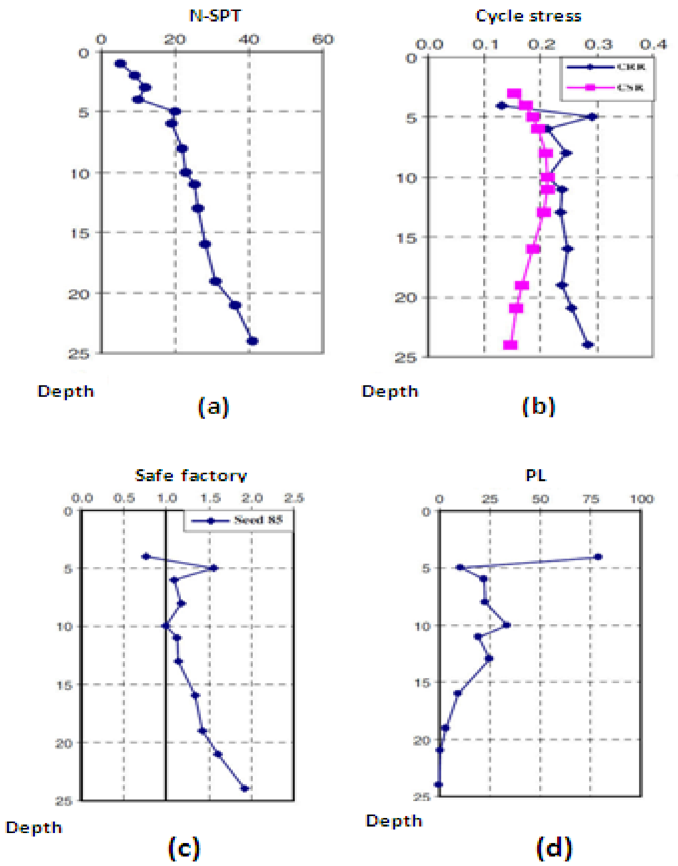


Figure 2: Sample Output of Cyclic Liquefaction Resistance and Probability of Liquefaction

With this profile, the engineer can determine which layers are sensitive to liquefaction from the viewpoint of an acceptable risk level. This advantage is also observed in Table 1. For example, in the case of 12 at the depth of10 m, comparison of calculated CSR and CRR suggests that there would be no liquefaction since CRR > CSR (albeit slightly). However, the field observation indicates the occurrence of liquefaction. The probability of liquefaction for this case is 32, which suggests that liquefaction may be possible. Similar observation was found in the case of 17, in which the deterministic approach shows a CRR of 0.16 and a CSR of 0.15, suggesting that liquefaction will not occur.

However, the field observation indicates the occurrence of liquefaction. For this case, the result of the probability analysis (PL = 38) does not yield a credible support of the occurrence of liquefaction; thus, the probabilistic analysis does not indicate an advantage over the deterministic method. Figure3 shows a plot of the factor of safety (FS)—defined as the simple ratio of CRR over CSR— against the probability of liquefaction for the 190log cases analyzed. It is observed that the PL value is very close to 0 if FS is greater than 2.0. Thus, a conservative and expensive design with a FS of 2 would almost eliminate the chance of liquefaction, even though it is recognized that the FS approach does not account for parameter and model uncertainties. However, if the FS is between 1.0 and 1.5, the effect of uncertainty on the computed FS becomes more significant—the probability of liquefaction can change drastically for a small change in the FS (Fig. 3). Confidence coefficient by liquefaction incidence probability for 190 data related to 19 bores (dipping rods) in the studied district are given in the fig-

ure. The relation between PL-Fs and correlation coefficient of =0.887 can be described as the following relation (equation 6).

$$P_L = \frac{1}{1+\left(F_S/0.783\right)^{6.63}} \tag{6}$$

The equation 5 shows that when PL approaches zero, confidence coefficient (Fs) doesn't exceed from 25.2, which is a conservative and expensive design. when Fs is between 1 through 1.5 (when Fs ranges 1 to 1.5) , uncertainty effect in the calculations becomes meaningful and liquefaction incidences probability changes effectively for small changes in Fs. For example, if PL < 15, the liquefaction doesn't crucially occur, which equals to confidence coefficient of 1.15.

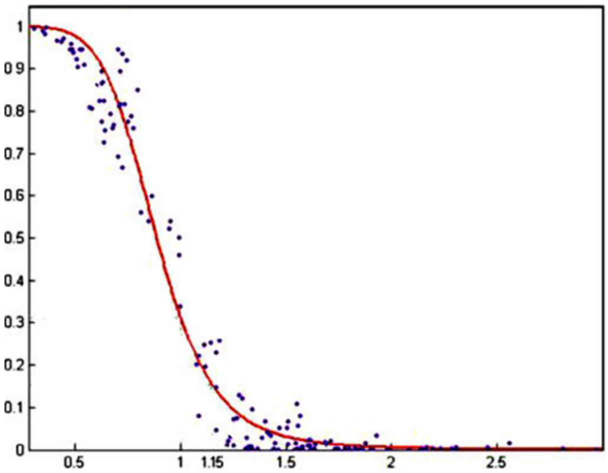


Figure 3: Factor of safety versus probability of liquefaction [15]

4 CONCLUSIONS

A new framework for the reliability analysis of liquefaction potential has been presented in this paper. Excellent results have been obtained in terms of being able to assess the liquefaction potential in a more rational way.

Regarding the performed comparisons between the deterministic method and probabilistic analysis-based method in this research, the efficiency of the probabilistic approach is well shown and it can be applied as a functional tool for application in engineering.

In this research, it was determined that confidence coefficient greater and smaller than 1 doesn't mean safety and/ or liquefaction in cadence for liquefaction, and for assuring liquefaction probability, reliability-based method analysis should be used. Regarding the proposed relationship between PL and FS, liquefaction probability of soil layers can be obtained by deterministic methods. This is a big advantage for geotechnical engineers who use common methods based on confidence coefficient for liquefaction potential assessment.

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